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UNUSUAL FOUNDATION CONDITIONS ENCOUNTERED ON THE CENTRAL AND SOUTHERN FLORIDA PROJECT

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ABSTRACT

Unusual foundation conditions encountered on the Central and Southern Florida Project include (1) very pervious solution-riddled limestones in the foundations of water-retention works, (2) thick layers of soft peat in levee foundations, and (3) structure foundations made up of interbedded sand and porous limestone.

Because of the nature of the project, which includes large shallow reservoirs bounded by low levees, prevention of seepage losses through the porous rock by grouting or cut-off walls is not economically feasible. Capacities of water-control works on the landside of the levees must be designed to include the underseepage flow. Quantitative determinations of the probable underseepage quantities have been based on permeabilities computed from the results of well-pumping tests. A limited number of discharge measurements made since completion of the levees show good agreement with computed seepage losses.

Unwatering of excavations for structures in the porous rock is very expensive and requires very large pumping capacities. No large excavations have yet been unwatered in the area of very pervious rock, and it is anticipated that special measures such as cofferdam grouting will be required in the unwatering of such excavations.

Very large embankment settlements must be provided for in designing levees on deep peat. For embankments 10 to 15 feet high, an allowance of about 50 percent of the thickness of the peat is required for settlement.

Practically all the peats encountered on the project have very low shearing strengths. The only shearing strength used in design is cohesion of about 0.1 tons per square foot.

Many spillways, pumping stations, and other structures to be built for the project must be founded on formations made up of thin layers of porous limestone underlain or interbedded with fine sand. Wherever possible, the structures are founded on rock. It is believed that a thin rock layer has value in that it acts as a foundation raft to reduce differential settlement. Great care must be exercised in excavating and unwatering foundations of this nature because of the possibility of causing damage by overblasting the rock or by inducing piping of the fine sand.

UNUSUAL FOUNDATION CONDITIONS ENCOUNTERED ON THE CENTRAL & SOUTHERN FLORIDA PROJECT

Paul H. Shea*

General

A large portion of the Central and Southern Florida Project is to be built in the grassy, swampy plain of the Everglades. The region is extremely flat, having an average slope from north to south of about a foot in five miles. Throughout the Everglades the surface soil is peat which ranges in thickness from a few inches to over 15 feet. Underlying the peat along many miles of the alinements of levees to be built for the project is a series of extremely pervious solution-riddled Pleistocene limestones about 50 feet thick. The limestones are the Miami oolite, which is only a few feet thick along most of the levee alinements, and the underlying Fort Thompson formation. In areas where the thick series of limestones is absent, the materials below the peat are interbedded sands, marls, and limestones.

The three major foundation conditions found on the project which are to a certain extent peculiar to the region are those briefly described above. To summarize, they are (1) the presence of extremely pervious rock in the foundations for water-retention works, (2) thick layers of very weak and compressible peat in embankment foundations, and (3) structure foundations made up of interbedded porous limestone and sand. Since one or more of these conditions exists at each levee or structure site, it may be said that all foundation conditions found on the project are somewhat out of the ordinary.

Permeable rock

The project provides for the retention of excess rainfall in conservation areas which are to be ringed with levees. Water thus stored would be used for irrigation when needed. Obviously, it would be very desirable to keep seepage losses from those areas at a minimum. Unfortunately, a large part of the retention reservoirs is underlain by very porous and permeable limestones which extend from at or near the ground surface to depths of 50 feet or more. The limestones are riddled with solution channels generally ranging from a fraction of an inch to 6 or 8 inches in diameter. No large cavities have been found in exploratory drilling for the project, although a few have been reported in previous investigations. The areal extent of the extremely permeable rock is indicated on figure 1.

The limestones in the Miami area are one of the most pervious formations ever investigated by the U. S. Geological Survey. That agency has found that the permeability of the limestones is comparable to that of a clean, well-sorted gravel. In the southeastern part of the south conservation area of the Central and Southern Florida Project, the permeability of the rocks is even higher than in the Miami area. Typical permeability values range from 60,000 to over 400,000 in the units most commonly used in soil mechanics of

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cm/sec x 10^{-4}. In the unit used by the U. S. Geological Survey, the permeabilities range from about 130,000 to 900,000. That unit is defined as: "The rate of flow in gallons a day, under prevailing conditions, through each foot of thickness of a given aquifer in a width of 1 mile for each foot per mile of hydraulic gradient."

Prevention or reduction of underseepage by grouting the porous rock or providing cut-off walls is not economically feasible. The cost of such treatment would be many times the cost of the levees. Canals, pumping stations, and control structures in the areas which will be affected by the seepage must be designed with capacities which will include the underseepage flow. A reasonably accurate estimate of the quantity of underseepage which will occur when water is stored in the conservation areas is required for the design of these structures. One of the most difficult features of the subsurface investigation for the project has been the devising and performance of tests to obtain the necessary estimates of foundation permeability and the interpretation of the results of the tests.

In December 1950 a series of well-pumping tests to determine foundation permeabilities was begun. Tests were performed in the usual manner by pumping from a central well and observing the drawdown produced in observation wells at various distances from the pump well. A rather unusual feature of the tests was that drawdowns were extremely small because of the high permeability and very accurate measurement was required. Electric point gages accurate to .001 foot were used for measuring drawdowns. Soon after the pumping tests began, it became apparent that none of the published methods for computing permeabilities from such tests was applicable to the physical conditions encountered. In the Everglades, the porous limestone is overlain by sawgrass peat of varying thickness. Throughout most of the area the uppermost portion of the rock is much less pervious than the remainder because of the sealing of pores and solution channels by peat, marl, or redeposited calcite. The thickness of the relatively tight layer varies from a few inches to 2 or 3 feet. The water surface in the Everglades almost always lies either above the ground surface or in the peat. In the vicinity of a pumping well, the water seeps downward through the dense top layer into the aquifer and then flows toward the pump well to replace water removed during a pumping test.

Practically all published methods for computing permeabilities of porous materials by means of field pumping tests are based on one of the following two assumptions: either (1) the aquifer extends upward to the ground surface, or (2) the aquifer lies beneath a layer of completely impervious material. It is also assumed that water removed during a pumping test is supplied by depletion of water stored in the aquifer or by a distant source, or by both. It is evident that those assumptions do not apply to pumping tests in the Everglades where water is supplied by vertical flow from above the aquifer through a layer of lower permeability. A further complication in performing the pumping tests and interpreting the results is the presence of horizontal beds of relatively impervious fresh-water limestone dividing the more pervious marine limestone into two or three aquifers. Wherever that condition exists, sets of piezometers must be installed in all aquifers to make the required determination of hydraulic gradients.

Details of the mathematical treatment required for the derivations of the necessary formulas for the computations of permeability and underseepage quantities for foundations of the type found in the Everglades are beyond the scope of this paper. Formulas for permeability and underseepage for the simplest case, that of a homogeneous aquifer with a blanket of relatively low

permeability, are shown on figure 2. The much more complicated cases of two- and three-aquifer systems with intervening blankets require very lengthy computations for their solution. Those methods are not reproduced herein.

Well-pumping tests have provided the only means of arriving at a quantitative answer to the problem of determining foundation permeabilities. Such tests are extremely expensive and only a limited number could be made. To extend the information from the pumping tests, so-called "recharge" tests have been made in core borings along the levee alinements and at structure sites. The procedure followed is to test the drill holes in 5-foot sections by pumping water into the holes at a rate which will maintain a constant head. The "take" in gpm per foot of head is used as a rough index of the permeability of the material tested. Attempts to compute actual permeabilities from recharge tests have been found to give misleading results.

Information from the pumping tests, recharge tests, geologic studies and other available data were utilized in making the required estimate of underseepage quantities. In the area of highest permeability, along levee 30 and the south end of levee 33 (see figure 1), it is estimated that the underseepage will be over 100 cfs per mile of levee for each foot of head acting. Underseepage in that area will not be directly proportional to the head because seepage flow will be turbulent at heads of over one foot, according to the data obtained from the pumping tests.

Since the completion of levees 30, 33, and 37 which are founded on very pervious rock, as indicated by figure 1, a few discharge measurements have been made in the continuous borrow pits on the landside of the levees. Those measurements have been made to determine the quantity of underseepage which is intercepted by the continuous landside borrow pit. Lines of piezometers have been installed to permit observation of the hydraulic gradients in the foundations. From those observations it is possible to compute the approximate percentage of the underseepage flow which is being picked up by the borrow pits. Opportunity for such measurements has been very limited because the manner in which the control structures have been operated has resulted in free flow in only one short stretch of borrow pit. That reach, which is the L-30 borrow pit between the Dade-Broward dike and the Miami canal, has been metered on several occasions. The agreement between the measured and predicted discharges has been very satisfactory. With head differentials of slightly less than 2 feet, underseepage has been about 200 cfs per mile.

The solution-riddled limestone is responsible for another very serious difficulty in the construction of the project--the extremely high cost of unwatering excavations. Wherever it is necessary to perform work in the dry below the water table, almost incredibly high rates of pumping are required. For example, in unwatering a small excavation for a culvert headwall at the intersection of levee 33 and the Miami canal, the contractor was forced to pump at a rate of over 20,000 gpm although the hole was completely ringed with steel sheet piling. The head acting was only 8 feet. Another example of the high permeability of the rock is furnished by experience at an experimental ring levee built near the south end of levee 30. The area surrounded by the ring levee was filled by pumping from the borrow pit in the course of investigations which were being performed. The borrow pit was L-shaped and about 100 feet wide by a total length of 600 feet measured on the centerline. Pumping from the pit at a rate of 10,400 gpm for over 30 hours caused no measurable drawdown on a recording gauge set near the mid-point of the area, although the total volume of water pumped would have filled a hole hav-

ing the area of the borrow pit to a depth of about 35 feet.

To date, no large excavations have been unwatered in the porous rock area. If foundations for major structures in that area must be unwatered at some future time, it is believed that extraordinary measures such as cofferdam grouting or cut-off walls extending to some relatively impervious bed will be required.

Peat

Methods of embankment construction in the Everglades are dictated by the physical conditions under which the levees must be built. Because of the presence of a layer of soft peat at the surface throughout almost the entire area, conventional methods using hauled-in fill and layer-compaction are impractical. Embankment materials are cast-in-place from continuous adjacent borrow pits by draglines supported on mats. In areas of very deep peat, mats alone are not sufficient to support the equipment and it is necessary to place a layer of sand or rock excavated from the borrow pit for a working base ahead of the machines. On several occasions, borrow pit banks have given way under the weight of the draglines causing the machines to slide into the borrow pit.

In designing levees on peat foundations, the first step is to determine the shearing strength of the peat. For almost all soils, sufficiently accurate values of shearing strength can be obtained from the results of direct shear tests. Peat is not a soil, however, as the following characteristics demonstrate. Typical qualities of Everglades peat are: dry unit weight, 5 to 10 pounds per cubic foot: natural water content, usually over 500 percent and often over 1,000 percent: loss on ignition, over 90 percent. A material which is over 90 percent plant fiber can hardly be called a soil. Direct shear tests on peat give completely erroneous results. The predetermined failure plane cuts across the fibers and the apparent angle of internal friction is usually between 20 and 25 degrees. When the same material is tested in a confined compression apparatus, where failure can occur on any plane, very low shearing strength is obtained. Angles of internal friction from triaxial tests are usually less than 5 degrees. Figure 2 shows the comparison between the results obtained by the two test methods on similar samples. Cohesion of about 0.1 ton per square foot is the only shearing strength considered in design.

Rather steep embankment slopes (1 on 2 and 1 on 3) have been adopted for all levees constructed on the project to date. In the areas of deepest peat, a few small slides have occurred during construction, but after repairing the failed areas, the completed levees have given no further trouble. The embankment materials are excavated under water and cast into the levees. They are completely saturated and provide a greater load on the foundation than will ever occur after the construction period.

In the design of a proposed levee, where the embankment is to be founded on very deep peat having very little strength, a mat of rock, sand, and marl is to be provided, extending some 30 feet outside the toes of the embankment, which will act somewhat as a "spread footing" for the levee. Stripping of the foundation will be limited to the cutting and removal of trees and large brush. From experience it has been found that the strength imparted by the surface root mat is of great value in supporting superimposed loads. Tractors equipped with special tracks can traverse the peat as long as the root mat is not ruptured, but bog down as soon as that support is destroyed. Some idea of the difficulties anticipated in building this levee may be gained from

the fact that construction by draglines was discarded as impractical because it is considered impossible to operate such equipment in the area. The embankment will be placed by hydraulic dredge and final shaping will be done by light draglines and bulldozers after sufficient fill has been placed to support their weight. The adequacy of the design has not yet been tested, since embankment construction will not begin for about 3 months.

From the very inception of the project, it has been recognized that large settlements of embankments founded on peat were inevitable. Early estimates, based both on experience in the construction of levees around Lake Okeechobee and on very incomplete consolidation test data, were that settlement would be equal to 50 percent of the peat thickness and that half of the settlement would occur during construction. Since the heights of the first levees built varied within a rather narrow range of about 10 to 15 feet, no correction was made for the varying load produced by different levee heights. Construction grades for the first levees built on the project were placed above final design grade by an amount equal to 25 percent of the peat thickness in accordance with the method used for estimating the settlement. Profiles of those levees run a year after completion show no significant change in elevation since completion of the embankments. Apparently, practically all the settlement occurred during construction.

Settlement plates were installed at many points along the levee alinements just before placing the fill. At each plate, the thickness of the peat was determined by auger borings. Elevations of the plates have been checked several times since the completion of the embankments. Although individual settlements differ considerably, the average of all the readings comes very close to the original estimate of 50 percent of the peat thickness. There has been very little change in the elevations of the plates after the first reading, confirming the evidence furnished by the postconstruction profiles that practically 100 percent of the settlement occurs during construction. While the average settlement has been about 50 percent of the peat thickness, the settlement is not independent of the levee height. Figure 3 is a curve showing the observed variation of the settlement with the height of embankment.

Use of consolidation tests for the prediction of settlement has been only partially successful. While the amount of settlement can be determined with fair accuracy, the predicted length of time required for the settlement is not even approximately correct. Where the consolidation tests indicate 15 to 20 years for 90 percent settlement, the actual settlement takes place in a few days or weeks. The reason for this discrepancy has not been investigated.

Interbedded rock and sand foundations

It will be necessary to build several fairly large structures on foundations which are made up of rock and sand. In some cases, the structures will be founded on beds of rock both overlain and underlain by sand, and in others on formations made up of alternating beds of consolidated and unconsolidated materials. To those who are not familiar with the geology of the coastal plain areas of the unglaciated portions of the United States, the occurrence of rock interbedded with and underlain by unconsolidated materials may appear strange, but such conditions are found throughout a large part of the Florida peninsula. Whenever possible, structures such as pumping stations and spillways will be founded on rock. Even if the rock layer is relatively thin, it is felt that it will act more or less as a large foundation raft which will minimize differential settlement. Cutting through the rock bed on which the structure rests will be avoided, if possible. In some cases, drains and

spillway apron sills may require excavation through the rock. Thin rock beds with essentially vertical solution channels are considered analogous to concrete slabs with weep holes. Continuous toe drains cut through the rock and into the sand will be provided at the downstream end of structures on such foundations to relieve the uplift pressure in the sand beds and prevent subsurface erosion of the sand. The drains will be filled with graded filter material and will contain perforated collector pipes which will be vented to tail water through riser pipes.

Some difficulty is anticipated in unwatering foundations made up of interbedded rock and sand. Pumping from sumps, which is common practice in rock foundations, could be done safely only if special precautions were observed. Concentration of flow into sumps would almost certainly cause movement of sand, and consequent damage to the foundation. Such movement could be prevented by filling the pump sumps with properly graded filter material and setting the pump intakes in perforated pipes. Continuous trenches through the rock bed on which the structure is to be founded could not be tolerated, and in many cases a large number of sumps would be required because of the lack of interconnection. All in all, it is believed that unwatering of heterogeneous foundations of the type found on this project can best be accomplished by the use of well-point systems. Such systems, when properly installed and operated, tend to stabilize the fine sands rather than rendering them "quick" and result in completed excavations having firm floors and side slopes.

Great care must be exercised in excavating for the foundations of structures to be placed on thin rock layers. In a massive rock foundation, overexcavation will result in added cost, but in the case of a foundation layer only a few feet thick, overexcavation can have disastrous consequences. It is entirely possible to destroy the foundation completely by excessive blasting. Blasting to grade a foot or two below the surface of a thin rock layer is impractical and should not be attempted. Rock removal by rooter, rock plow, air spade, or some other easily controllable method is required.

Effect on design and construction

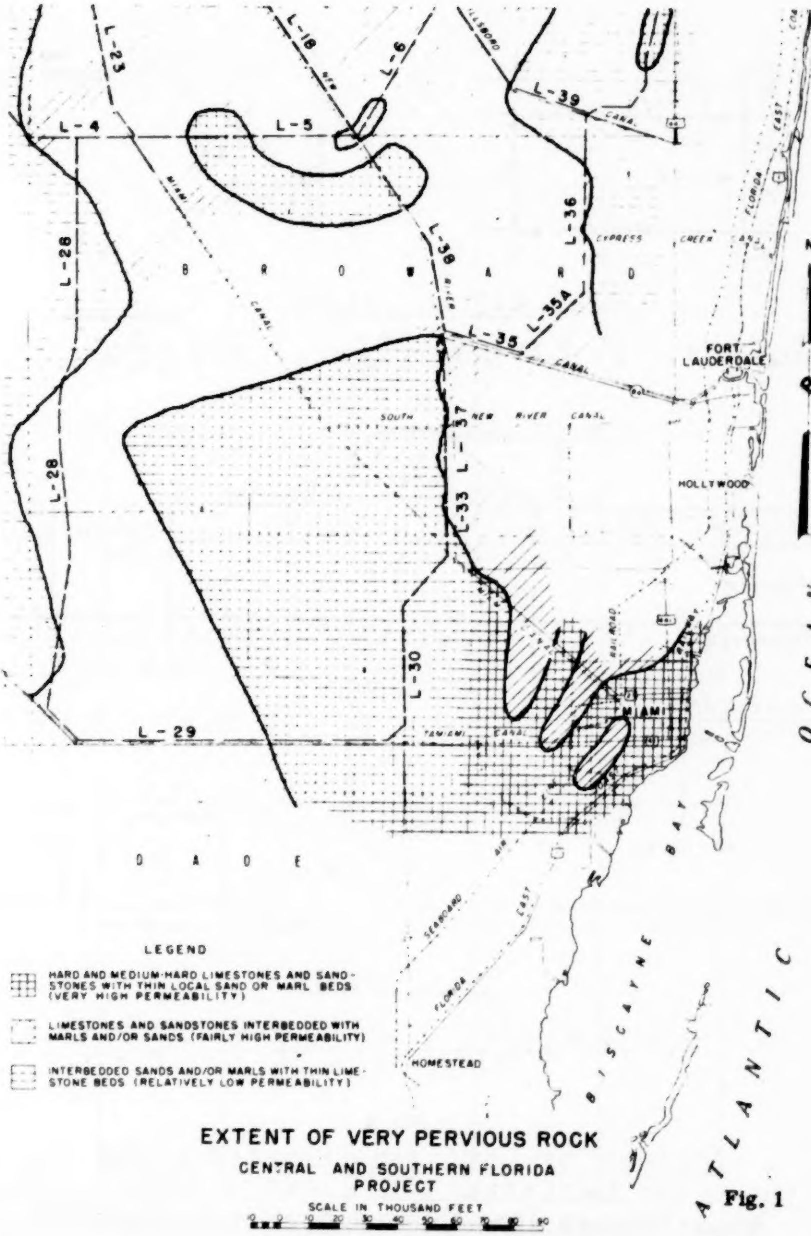
The unusual conditions which have been described above present problems in design and construction, some of which have no ready solution. Large seepage losses and unusually great settlements of embankments are difficulties which can be allowed for but can not be prevented without the expenditure of much more money than the benefits gained would warrant. Very high unwatering costs are inevitable and must be considered in estimating the cost of proposed structures.

All structures are designed for 100 percent uplift over the full base area. Wherever any possibility of piping or flotation of foundation sand exists, drainage facilities are provided to relieve the uplift pressures. Structures proposed for the project are relatively small and foundation loads rarely exceed 2 or 3 tons a square foot. The rocks and sands on which the structures will be founded usually have more than adequate bearing capacity for the loads. Pile foundations have not been required for any of the structures designed to date.

Careful inspection of foundation excavation work is required to prevent damage to foundations by improper unwatering or excavation procedures. Wherever other considerations will permit, foundation grades are raised to the highest possible level to obtain the maximum thickness of rock under the structures and reduce concrete quantities.

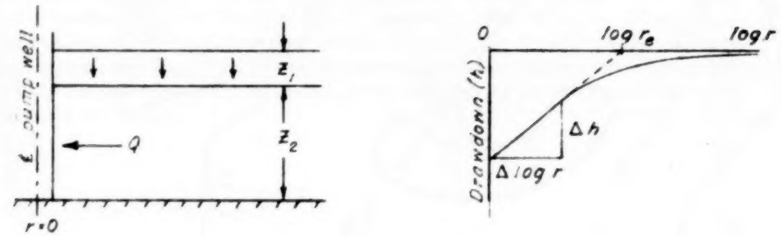
Summary

Very porous rock, thick beds of soft peat, and foundations made up of interbedded rock and sand complicate the design and construction of levees and structures for the Central and Southern Florida Project. The design and construction of engineering works adequate for their intended purpose can be accomplished in spite of the unfavorable foundation conditions, but only by recognizing the foundation problems and making the necessary allowances in design.



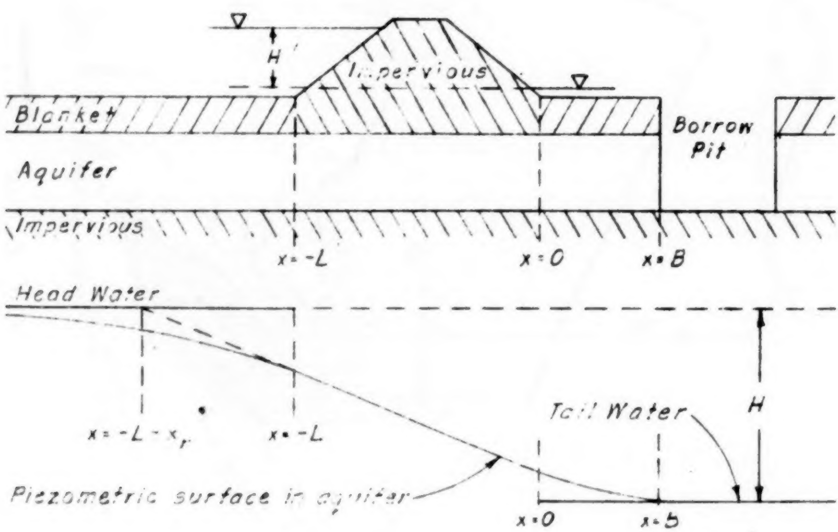
EXTENT OF VERY PERVIOUS ROCK
CENTRAL AND SOUTHERN FLORIDA PROJECT

Fig. 1



RADIAL FLOW

$a r_e = 1.123$; $Q = 2.73\, k_2\, z_2 \frac{\Delta h}{\Delta \log r}$; $a = \sqrt{\frac{k_1}{k_2 z_2 z_1}}$

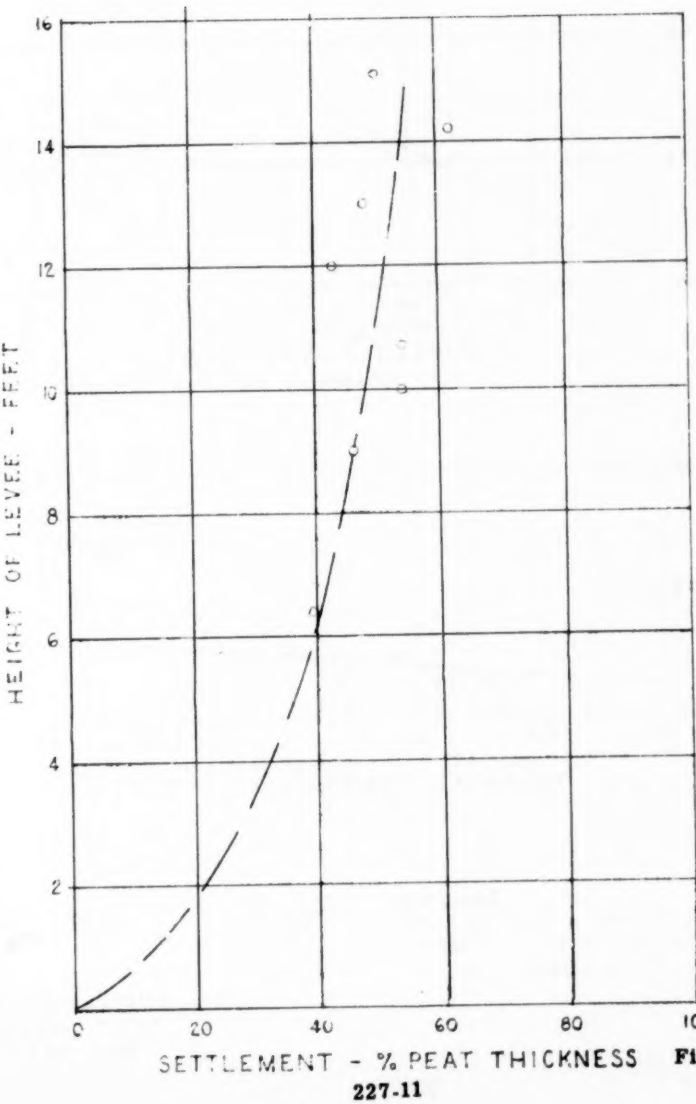


SHEET FLOW

$x_r = \frac{1}{a}$; $Q \approx \frac{k_2 z_2 H}{L + x_r + R}$, where $R = B$ if $B \le x_r$,
$R = x_r$ if $B \ge x_r$.

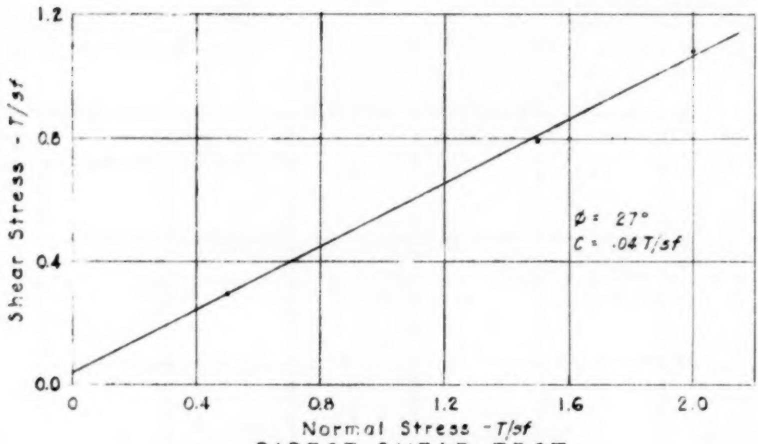
Fig. 2

METHOD OF COMPUTING PERMEABILITIES
AND UNDERSEEPAGE QUANTITY FROM
WELL-PUMPING TEST (SINGLE BLANKETED AQUIFER)

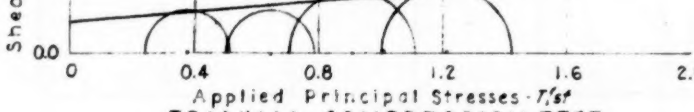


SETTLEMENT - % PEAT THICKNESS **Fig. 3**

227-11



DIRECT SHEAR TEST
(SAMPLE A)



TRIAXIAL COMPRESSION TEST
(SAMPLE B)

	Sample A	Sample B
Dry unit wt.	10 lbs./cf	8 lbs./cf
Water content	462%	655%
Loss on ignition	90.7%	91.3%

Fig. 4

TYPICAL SHEAR TEST CURVES FOR PEAT

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